IS 7784 (Part 1) : 2013

(Reaffirmed 2019)

भारतीय मानक

आर पार जल निकास कार्यों के डिजाईन — रीति संहिता

भाग 1 सामान्य लक्षण

(दूसरा पुनरीक्षण)

Indian Standard

DESIGN OF CROSS DRAINAGE WORKS — CODE OF PRACTICE

PART 1 GENERAL FEATURES

(Second Revision)

ICS 93.160

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July 2013 **Price Group 7**

Canals and Cross Drainage Works Sectional Committee, WRD 13

FOREWORD

This Indian Standard (Part 1) (Second Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Canals and Cross Drainage Works Sectional Committee had been approved by the Water Resources Division Council.

Cross drainage works are structures which are constructed to negotiate an aligned carrier channel/canal over, below or at the same level of a drainage or another carrier channel/canal.

This standard was first published in 1975 and revised in 1993.

This revision has been taken up in the light of the comments received from the members. The salient changes made in this revision are as follows:

a) Table indicating losses for type of transition has been deleted and values for coefficient of expansion or contraction for various transitions (open and closed) has been added.
b) Separate sub clauses on effect of channel sinuosity, checks, flumes (measuring structures), and hydraulic jump have been added under various losses due to different factors.

This Code is formulated in two parts, Part 1 deals with general features and Part 2 with specific requirements, for various types of cross drainage works.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

DESIGN OF CROSS DRAINAGE WORKS — CODE OF PRACTICE

PART 1 GENERAL FEATURES

(Second Revision)

1 SCOPE

This standard (Part 1) covers general features pertinent to the design of various types of cross drainage works and incorporates investigations and studies connected therewith.

2 REFERENCES

The standards listed below contain provisions which, through reference in this text constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards listed below:

IS No.	*Title*
1893 : 1984	Criteria for earthquake resistant design of structures (*fourth revision*)
2912 : 1999	Liquid flow measurement in open channels — Slope area method (*first revision*)
2951	Recommendation for estimation of flow of liquids in closed conducts:
(Part 1) : 1965	Head loss in straight pipes due to friction resistance
(Part 2) : 1965	Head loss in valves and fittings
4410	Glossary of term relating to river valley projects:
(Part 11/Sec 5) : 1977	Hydrology, Section 5 Floods
(Part 15/Sec 5) : 1992	Canal structures, Section 5 Cross drainage works (*first revision*)
5477 (Part 4) : 1971	Methods for fixing the capacities of reservoirs: Part 4 Flood storage
7894 : 1975	Code of practice for stability analysis of earth dams
8237 : 1985	Code of practice for protection of slope for reservoir embankments (*first revision*)
10751 : 1994	Planning and design of guide banks for alluvial rivers — Guidelines (*first revision*)
11532 : 1995	Construction and maintenance of river embankments (levees) — Guidelines (*first revision*)
12094 : 2000	Guidelines for planning and design of river embankments (levees) (*first revision*)

3 TERMINOLOGY

For the purpose of this standard, the definitions given in IS 4410 (Part 11/Sec 5) and IS 4410 (Part 15/Sec 5) shall apply.

4 CATEGORIES OF CROSS DRAINAGE WORKS

Cross drainage works can be classified under the three broad categories listed at **4.1** to **4.3**, based on the type of the structure to negotiate a canal over, below or at the same level of the drainage channel.

4.1 Structures for Canal Over a Natural Drainage Channel

The structures falling under this category are aqueducts, syphon aqueducts and culverts. Maintenance of structures in this category is relatively more convenient, as these are generally above the ground and hence open for inspection.

4.2 Structures for Canal Underneath a Natural Drainage Channel

The structures falling under this category are superpassages and syphons including well syphons. In case of syphons the maintenance is difficult as these run below the natural drainage channel and are, therefore, not easily accessible to inspection.

4.3 Structures for Canal Crossing a Natural Drainage Channel at the Same Level

Structures falling under this category are level crossings and inlets, with or without escapes.

NOTE — Wherever the word ‘canal’ is used, it should be meant as canal/carrier channel.

5 SELECTION OF THE TYPE OF CROSS DRAINAGE WORK

5.1 While aligning the canal, the type of cross drainage work envisaged should always be kept in view. The economics of various types of cross drainage works *vis-a-vis* alternative alignments should be considered before deciding upon the site and type of crossing. As a general guide, for deciding upon the type of the cross drainage work, important considerations are as given below:

a) Full supply level and functions of canal — *vis-a-vis* high flood level of the drainage channel;
b) Topography of terrain;
c) Regime of the stream;
d) Foundation strata;
e) Dewatering requirements;
f) Ratio of design flood to be provided in drainage channel to the discharge in the canal; and
g) Envisaged head loss.

5.1.1 *Full Supply Levels of Canal* vis-a-vis *High Flood Level (HFL) of Drainage Channel*

The choice of any particular type of cross drainage work is dependent on the high flood level (HFL) in the drainage channel to be negotiated. Aqueducts are generally proposed when the bed level of canal is well above the HFL of the drainage channel. Super passages are generally proposed when the full supply level (FSL) of the canal is well below the bed level of the drainage channel. When the bed level of the canal is at, or below, the HFL of the drainage channel, the depression of the bed of the drainage channel is often a more economical proposal and in such cases syphon aqueducts may be considered.

5.1.2 *Topography of Terrain*

Detailed examination of the topography of the terrain is essential to locate a stable reach of the drainage channel with good foundations permitting, preferably, a right-angle crossing. Topography of the terrain may also permit diversion of one channel into another and locating the cross drainage work below the confluence of the two channel for greater economy.

5.1.3 *Regime of Drainage Channel*

The regime of a drainage channel requires careful examination. For drainage channel carrying high sediment charges or drift materials, the possibility of choking up of the syphon and the effect of fluming of the drainage channel should be kept in view.

5.1.4 *Foundation Strata*

The selection of the most suitable site and a good design, for any cross drainage work is intimately related to the engineering properties of the foundation, sub-strata at various alternative sites. These properties have, therefore, to be determined by site explorations. Where an alternative site, meeting other criteria, is available, the final choice would obviously depend on the location where the sub-strata available close to the bed of the stream is firm.

5.1.5 *Dewatering Requirements*

In the execution of foundation works for cross drainage structures dewatering of foundations may pose serious problems. An accurate estimate of the cost and procedure of dewatering requires to be carefully worked out when designs involve laying of foundations below the ground water table.

5.1.6 *Ratio of Design Flood in Drainage Channel to the Discharge in Canal*

Negotiating a canal below the drainage channel is generally more difficult and involves more head loss. However, if the topography and other features warrant a choice to be made between canal syphon and syphon aqueduct, then canal syphon may be preferred, only if the ratio of canal discharge to the design flood is substantially low.

5.1.7 *Envisaged Head Loss*

The choice of any particular type of cross drainage work is also dependent on the head loss that can be permitted in the canal. Whereas higher head loss can throw some area out of command, restriction on head loss may necessitate provision of wider sections making the structure costly.

6 DATA REQUIREMENT

6.1 For any type of cross drainage work some data is required which is common to all types of cross drainage works. A location map for the work with results of sub-surface exploration conducted at site, cross-sections of the stream, upstream and downstream of the proposed site, should be prepared, as given in **6.2** to **6.9**.

6.2 An index map to a suitable scale showing the recommended location of the cross drainage structure, the alternative sites of crossings investigated and rejected, the existing communications, the general topography of the country and the important habitations in the vicinity.

6.3 A catchment area map to a suitable scale, with contour markings at suitable intervals showing the main drainage channel from its sources together with all its tributaries. The map should also show the various locations of raingauge stations, gauging sites, etc, as also the general soil types and land use (that is forests, cultivated and uncultivated areas). The hydrological

observation sites should also be marked. Existing, under construction or proposed embankments and flood management measures should also be shown.

6.4 A detailed survey plan of the drainage channel to suitable scale showing important topographical features extending considerable distances, downstream and upstream, of the proposed site of crossing and either of its banks.

6.5 A site plan to a suitable scale showing details of the site selected and extending upstream and downstream, of the centre line of the proposed crossing and covering its approaches to sufficient distances, so as to demarcate levels, cadestral survey plot numbers, important topographical features like depressions near the proposed alignment of canal, general sub-soil water levels (with slope, if possible), etc.

6.5.1 The other requirements for the plan at **6.5** are:

a) Reference to the position of the bench-mark used as datum with its full description and reduced level;
b) Lines and identification numbers of the cross-sections and longitudinal sections of drainage channel taken within the scope of site plan and exact locations of their extreme points;
c) Locations of the various trial pits and/or borings with their identification numbers;
d) Contour of the drainage channel at intervals between 0.5 m to 1.5 m depending upon the terrain. This interval may be greater in mountainous regions;
e) Direction of flow of water;
f) Angle of direction of crossing; and
g) Cross alignment of canal further upstream for some distance beyond the limits of cross drainage works.

6.6 A cross-section of the drainage channel at the proposed site of the crossing to appropriate vertical and horizontal scales indicating the following information:

a) Cross-section covering the bed and banks of the channel portion and the ground levels beyond the banks covering the entire flood plane, or from ridge to ridge at close intervals to sufficient distances on either side showing all uneven features and habitations, if any;
b) Nature of the soil in bed, banks and approaches, with trial pit or bore-hole sections showing the levels and natures of the various strata down to stratum suitable from foundation considerations and from considerations of safe bearing capacity of soil;
c) Low-water level; and
d) Maximum flood level.

6.7 Longitudinal section of the drainage channel covering a reasonable reach to suitable scale, showing the location of the cross drainage work, with levels of the observed flood, the low water and the bed levels at suitably spaced intervals along the line of the deep water channel.

6.8 A note giving the salient features relating to the catchment area, the meteorological conditions experienced thereon, besides the following other points:

a) Any predictable (future) alteration in the land use;
b) Storages in the catchment (artificial or natural) and embankment breaches that have occurred in the past;
c) Short duration intensity and frequency data in respect of rainfall in the catchment;
d) Liability of the site to seismic disturbances;
e) Likelihood of heavy sediment charge or floating timber;
f) Particulars of foundation exploration data incidental to design requirements; and
g) Recuperation tests, where foundation depth is more than 3 m below the water table and where the strata are pervious.

6.9 A note giving the salient design features of structures existing upstream or downstream of the proposed site.

6.9.1 Presence of dams, barrages, weirs, etc, on the natural drainage channel in the vicinity either upstream or downstream, may affect the hydraulic characteristics of the natural drainage channel, like obliquity and concentration of flow, scour, silting of bed, change in bed levels, flood levels, etc. These effects should be considered in the design of the cross drainage work.

6.10 For preparing the design of a cross drainage structure, the following specified hydraulic data should also be made available.

6.10.1 *Canal*

a) Full supply discharge, Q;
b) Bed width;
c) Full supply depth;
d) Water surface slope;
e) Bed level;
f) Bed slope;
g) Full supply level;
h) Top of bank level;
j) Cross-section of canal showing natural ground level;

k) Sub-soil water level; and

m) Nature of bed material and value of '*n*' (rugosity coefficient in Manning's formula).

6.10.2 *Drainage Channel*

a) Extent and nature of drainage area (catchment area);

b) Maximum annual rainfall and the period (year) of data;

c) Maximum intensity of rainfall with year;

d) Maximum observed flood discharge at the site;

e) Maximum flood level;

f) Water surface slope;

g) Site plan of proposed crossing including contours;

h) Log of borehole or trial pit data;

j) Type of bed load of drainage channel;

k) Longitudinal section of the stream for suitable distance upstream and downstream of the canal depending upon site conditions;

m) Cross-section of the drainage channel for a distance of 100 m to 300 m upstream and downstream, at intervals of 10 m to 50 m;

n) Waterways provided in road and railway bridges or other hydraulic structures on the drainage channel;

p) Spring water level at the crossing site in May and October; and

q) Silt factor.

7 DESIGN FLOOD FOR DRAINAGE CHANNEL

7.1 Design flood for drainage channel to be adopted for cross drainage works should depend upon the size of the canal, size of the drainage channel and location of the cross drainage. A very long canal, crossing a drainage channel in the initial reach, damage to which is likely to affect the canal supplies over a large area and for a long period, should be given proper weightage.

7.2 Cross drainage structures are divided into four categories depending upon the canal discharge and drainage discharge. Design flood to be adopted for these four categories of cross drainage structures is given in Table 1.

7.3 Where possible, the discharges determined by different methods mentioned in IS 5477 (Part 4) should be compared to see if any large variations are exhibited and the most reasonable value, giving weightage to the one based on observed data, should be adopted. Where there are cross drainage works already existing on the same drainage channel, full data regarding the observed flood should be obtained and the new cross drainage works designed, with such modifications in the design flood as may be considered necessary.

7.4 To safeguard against unforeseen nature of flood intensities the foundation of the cross drainage structure should be checked for a check flood discharge of value 20 percent higher than the design flood given in Table 1.

8 HYDRAULIC DESIGN ASPECTS

8.1 Waterway

8.1.1 Waterway for a cross drainage work is fixed from hydraulic and economic considerations with particular reference to,

a) design flood;

b) topography of the site;

c) existing and proposed section and slope of the

Table 1 Design Flood Values

(*Clauses* 7.2 *and* 7.4)

Sl No.	Category of Structure	Canal Discharge m^3/s	Estimated Drainage Discharge[1)] m^3/s	Frequency of Design Flood
(1)	(2)	(3)	(4)	(5)
i)	A	0-0.5	All discharges	1 in 25 years
ii)	B	0.5-15	0-150	1 in 50 years
			Above 150	1 in 100 years
iii)	C	15-30	0-100	1 in 50 years
			Above 100	1 in 100 years
iv)	D	Above 30	0-150	1 in 100 years
			Above 150	(*see* Note 2)

NOTES

1 The design flood to be adopted as mentioned in this table should in no case, be less than the observed flood.

2 In case of very large cross drainage structures where estimated drainage discharge is above 150 cumecs and canal design discharge is more than 30 cumecs, the hydrology should be examined in detail and appropriate design flood adopted, which should in no case be less than 1 in 100 years flood.

[1)] This refers to the discharge estimated on the basis of river parameters corresponding to maximum observed flood level.

drainage channel in the vicinity of the crossing;

d) permissible afflux; and

e) construction and maintenance aspects.

8.1.2 In plains, the drainage channels are generally in alluvium and the waterway usually provided in works without rigid floor is about sixty to eighty percent of the perimeter, given by Lacey's formula:

$$P_w = C\,[Q]^{1/2}$$

where

P_w = wetted perimeter, in m;

C = coefficient varying from 4.5 to 6.3 according to local conditions, the usual value adopted being 4.8 for regime channel; and

Q = design flood, in m^3/s.

8.1.2.1 The value of wetted perimeter obtained from **8.1.2** is the total waterway between the two faces of the abutments.

8.1.2.2 In works with rigid floors, however, waterway can be further flumed within the permissible limits of velocity negotiated through the available ventages. Ordinarily such velocities should be limited to the values given in Table 2.

8.1.3 For sub-mountainous and mountainous terrains with flashy flows, the waterway is provided within the width of the existing stream. Where the slope of the natural drainage channel is quite steep suitable methods maybe adopted to bring the velocity within the desired limits.

8.1.4 The minimum dimension of openings should be such as to permit, as far as possible, manual clearing of deposits therein.

8.2 Clearance for Aqueducts

8.2.1 *Rectangular Openings*

The clearance shall depend upon the relative levels of the canal bed and high flood levels of the drainage channel. Values given in Table 3 are suggested as suitable minimum clearances (taking into account allowable afflux) for purposes of design, where available.

8.2.1.1 If the minimum clearances specified in Table 3 are not available; safety of the superstructure should be ensured against likely repercussions.

8.2.2 *Arch Openings*

Minimum clearance measured to the crown of the arch should normally be given as recommended in Table 4.

8.2.3 In the case of drainage channels, where a bed rise due to progressive silting is anticipated, the permissible clearance specified in Table 4 should be increased to allow for such aggradations depending upon the extent of silting.

8.2.4 *Free Board*

On aqueduct structures, the free board is reckoned from the high flood level (including afflux) in case of drainage channel and from the full supply level in case of canals, to the formation level of guide bank or canal embankment. The free board should not be less than 900 mm. Wherever heavy wave actions are anticipated, the free board should be suitably increased.

Table 2 Maximum Permissible Velocities

(*Clause* 8.1.2.2)

Sl No.	Types or Floors	Maximum Permissible Velocity m/s
(1)	(2)	(3)
i)	Metals face (steel and cast iron lined)	10
ii)	Face of concrete grade M 30 and above,	6
	Grade below M 30	4
iii)	Stone masonry face with cement pointing	3
iv)	Stone masonry face with cement plaster	4
v)	Brick masonry face with cement plaster	2.5
vi)	Brick masonry with cement pointing	2
vii)	Hard rock	4
viii)	Murum	1.5-2.0
ix)	Soil silt	0.7-1

NOTES

1 When the flow carries abrasive materials with it, the permissible values may be further reduced by 25 percent.

2 Hard steel troweling, power floating, smooth surface finish and continuous long curing can have higher abrasion resistance and higher velocities than that given in this table can be permitted for surface using cement.

Table 3 Minimum Vertical Clearances for Rectangular Openings
(*Clause* 8.2.1)

Sl No. (1)	Design Flood m^3/s (2)	Minimum Vertical Clearance mm (3)
i)	Below 3	450
ii)	3 and above but below 30	600
iii)	30 and above but below 300	900
iv)	300 and above but below 3 000	1 200
v)	3 000 and above	1 500

Table 4 Minimum Clearances for Arch Openings
(*Clauses* 8.2.2 *and* 8.2.3)

Sl No. (1)	Arch Opening, m (2)	Clearance (3)
i)	Less than 3	Rise or 0.8 m, whichever is more
ii)	3 and above but less than 6	2/3 rise or 1.0 m, whichever is more
iii)	6 and above but less than 21	2/3 rise or 1.25 m, whichever is more
iv)	21 and above	2/3 rise or 1.5 m, whichever is more

8.3 Clearance for Superpassages

8.3.1 *Clearance*

Clearances of about 50 percent of those recommended in **8.2.1** and **8.2.2** *mutatis mutandis* may be provided in case of superpassages.

8.3.2 *Free Board*

Free board recommended in **8.2.4** may be provided.

8.4 Afflux

8.4.1 The afflux to be adopted in the design should be such that which would correspond to the design flood.

8.4.2 The afflux should be restricted to such a value that the resulting velocity does not cause serious bed scour in the drainage or does not create submergence which cannot be permitted.

8.4.3 The effect of afflux on the submergence of the surrounding country should be specially studied.

8.4.4 The afflux may be calculated by either of the methods given at **8.4.4.1** and **8.4.4.2**.

8.4.4.1 *Rational formulae*

Broad crested weir discharge formula or orifice discharge formula depending upon the flow conditions through the cross drainage work openings, may be applied for calculating afflux. When the performance of the cross drainage work openings remains unaffected by the depth downstream of the obstruction, that is, a standing wave is formed, weir formula is applicable, otherwise the orifice formula holds good. Approximately, when the downstream depth D_d above the crest is more than eighty percent of the upstream depth D_u the Weir formula does not hold good.

a) *Weir formula*:

$$Q = 1.70\, C_w L H^{3/2}$$

where

Q = discharge through the openings, in m^3/s;

C_w = coefficient of discharge accounting for losses in friction; the values may be taken as under:

	Condition	*Value*
1)	Narrow openings with or without floors	0.94
2)	Wide openings with floors	0.96
3)	Wide openings without floors	0.98

L = linear waterway, in m;

H = total energy head upstream of the obstruction in m, that is, $D_u + v^2/2g$;

D_u = depth of flow upstream in m;

$v^2/2g$ = velocity head where v is the average velocity in the approach section worked out from the known width (W) of unobstructed section; and

W = width of unobstructed section.

b) *Orifice formula*

$$Q = C_0 \left[2g\right]^{1/2} L D_d \left[\frac{h + (1+e)\, v^2}{2g}\right]^{1/2}$$

where

Q = discharge through the opening, in m^3/s;

C_0 = coefficient of discharge;

g = acceleration due to gravity, in m/s^2;

L = linear waterway, in m;

D_d = depth downstream of the obstruction, in m;

h = afflux, in m;

e = factor accounting for recovery of some velocity as potential head on emergence from the cross drainage work openings; and

v = average velocity in approach section, in m/s.

The value of 'C_0' and 'e' to be adopted are given in Fig. 1 and Fig. 2. The afflux can be calculated knowing (a) the discharge, (b) the unobstructed width of the stream, and (c) the average depth downstream of the cross drainage work opening.

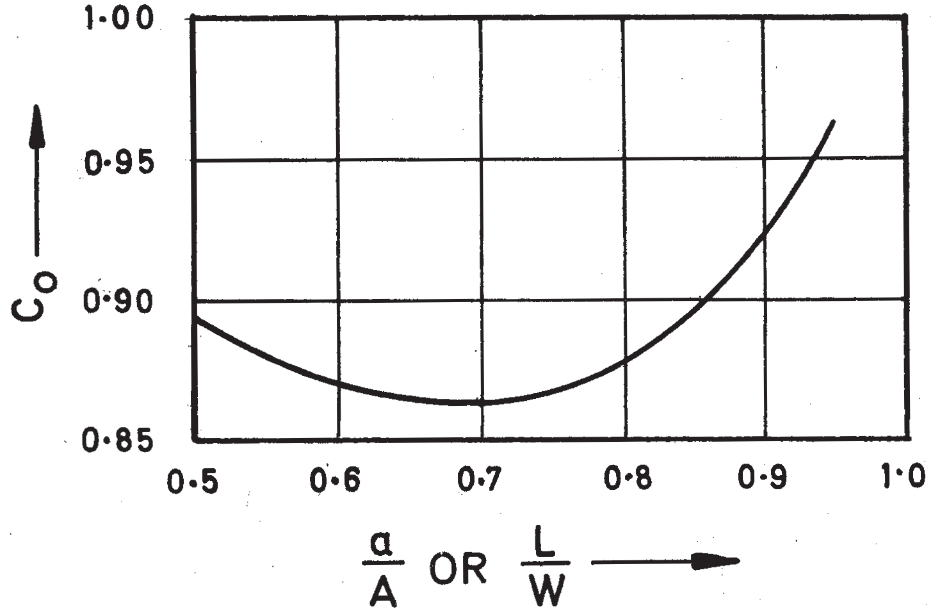

Fig. 1 Coefficient 'C_0' in the Orifice Formula

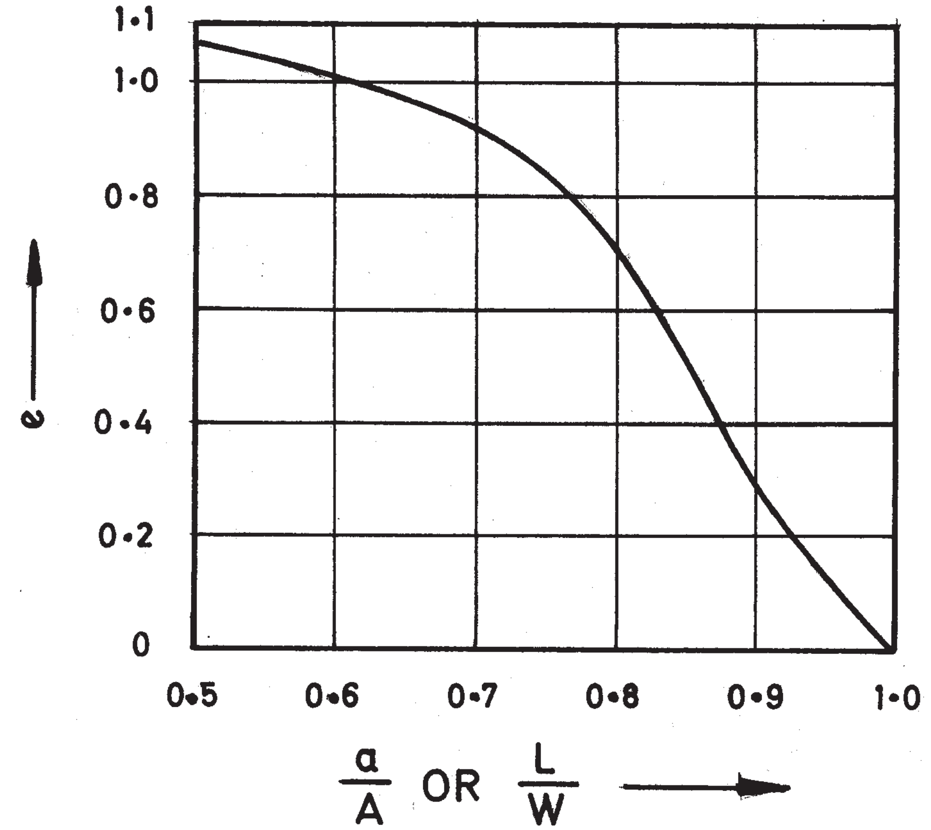

Fig. 2 Coefficient 'e' in the Orifice Formula

8.4.4.2 *Empirical formula*

When the area of obstruction is not very large compared to the original unrestricted area, the following formula gives reasonably good results:

$$h = \left[\frac{V^2}{17.85} + 0.015\,2\right]\left[\frac{A^2}{a^2} - 1\right]$$

where

h = afflux, in m;

V = velocity in the unobstructed drainage channel, in m/s;

A = unobstructed sectional area of the drainage channel, in m^2; and

a = sectional area of the drainage channel provided in the construction, in m^2.

If the value of V varies considerably in the unobstructed cross-section of the drainage channel, as in the case of a drainage channel which spills over its banks, V for the purposes of this formula may be taken as the average velocity in the main channel and correspondingly the value of A should be determined by dividing the total discharge by V.

8.4.4.3 In case of readily erodable beds, full afflux as calculated from **8.4.4.1** or **8.4.4.2** may not occur.

8.5 Depth of Scour

8.5.1 *Mean Depth of Scour*

The mean depth of scour, in metres, below the check/high flood level may be calculated from the equation:

$$d_{sm} = 1.34\left[\frac{D_i^2}{K_{sf}}\right]^{1/3}$$

where

D_i = discharge, in cumecs /m width. The value of D_i should be the maximum of the following:

a) Design flood divided by the effective linear waterway between abutments or guide bunds, as the case may be.

b) Value obtained should take into account any concentration of flow through a portion of the waterway assessed from the study of the cross-section of the drainage channel. Such modifications of the value may not be deemed applicable to minor cross drainage structures with overall waterway less than 60 m.

c) Actual observation, if any.

K_{sf} = Silt factor for representative sample of the bed material obtained up to the level of the deepest anticipated scour and given by the expression 1.76 $[d_m]^{1/2}$.

where

d_m = weighted mean diameter, in mm.

NOTES

1 d_m may be taken as the grain size at 50 percent passing from grain size distribution curve.

2 The above method of estimating d_{sm} is based on Lacey's theory for regime conditions in alluvial beds.

8.5.2 *Maximum Depth of Scour for Design of Foundation*

The maximum depth of scour below the highest flood level (HFL) at obstructions and configuration of the channel should be estimated from the value of 'd_{sm}' on the following basis:

a) For the design of piers and abutments located in a straight reach and having individual foundations without any floor protection works:

1) In the vicinity of piers : 2.00 d_{sm}

2) Near abutments : 1.27 d_{sm} approach retained
2.00 d_{sm} scour all around

b) For the design of floor protection works, for raft foundations or shallow foundations, the following scour values should be adopted:

1) in a straight reach : 1.27 d_{sm}

2) at a moderate bend : 1.50 d_{sm}

3) at a severe bend : 1.75 d_{sm}

4) at a right angled bend : 2.00 d_{sm}

NOTE — The values of scour depth obtained as above may be suitably modified where actual observed data is available.

8.6 Loss of Head (Energy Loss)

When water flows through any structure there are head losses due to various factors mentioned in **8.6.1** to **8.6.4**. The total loss of head occurring for a flow is represented as the sum of these losses as applicable Thus, if the total loss of head is denoted by H then

$$H = h_1 + h_2 + h_3 + h_4$$

where

h_1 = losses at the inlet and outlet (for syphon);

h_2 = losses at elbows or bends (for barrel);

h_3 = losses due to transitions (other than syphon); and

h_4 = losses due to skin friction (for barrel and trough).

8.6.1 *Loss of Head at the Inlet and at the Outlet of Syphons*

The formula for the losses at the entrance may be taken as:

$$h_1 = [1 + f_i]\frac{v^2}{2g}$$

where

h_1 = loss of head at entrance or at exit, in m;

f_i = coefficient which provides for the loss of head on entry. It may be taken, for all practical design purposes, as 0.08 for a bell mouth entrance and as 0.505 for cylindrical entrance with sharp edges (unshaped mouth of the same sectional area of the barrel);

v = velocity in syphon, in m/s; and

g = acceleration due to gravity, in m/s^2.

8.6.2 *Loss of Head Due to Elbows or Bends in Barrels*

The loss of head due to elbows or bends h_2, may be computed in accordance with the procedure given in IS 2951 (Part 2).

8.6.3 Well designed inlet and outlet transitions are necessary at the upstream and downstream approaches of cross drainage works. Following estimates of losses in the transitions h_3 generally hold for normal design and installation conditions. These are not applicable to syphons as for them this aspect is covered in **8.6.1**.

These losses exclude losses covered by introduction of trash racks on upstream approaches.

Transitions are generally used at the inlet and outlet of structures and where changes occur in the water section. An accelerating water velocity usually occurs in inlet transitions and a decelerating velocity in outlet transitions. The most common types of open transitions are the streamlined warp, straight warp, and broken back (*see* Note).

The depth of submergence of the top of the syphon opening is known as the water seal. For minimum hydraulic loss, a seal of 1.5 Δh_v, (where Δh_v is difference in velocity head), with a 76 mm minimum at the inlet headwall measured from the upstream water surface and no submergence at the outlet headwall should be provided. This submergence helps to prevent air and vortex problems. Outlet transitions should have no submergence of the opening in the headwall. If the submergence exceeds one-sixth of the depth of the opening at the outlet, the hydraulic loss should be computed on the basis of a sudden enlargement rather than as an outlet transition. The hydraulic loss, h_3, in a transition will depend primarily on the difference between the velocity heads at the open end of the transition and at the normal to centerline section of the closed conduit at the headwall. It is calculated using the equation:

$$h_3 = \frac{K\left(v_1^2 - v_2^2\right)}{2g}$$

where

v_1 = velocity of flow before the transition, in m/s;

v_2 = velocity of flow after the transition, in m/s;

g = acceleration due to gravity, in m/s^2 ; and

K = coefficient of expansion or contraction.

The following tabulation gives 'K' design values for various transitions:

Sl No.	*Type of Transition*	*Inlet*	*Outlet*
i)	*Open transition*		
	a) Streamlined warp to rectangular opening	0.1	0.2
	b) Straight warp to rectangular opening	0.2	0.3
	c) Straight warp with bottom corner fillets to pipe opening	0.3	0.4
	d) Broken back to rectangular opening	0.3	0.5
	e) Broken back to pipe opening	0.4	0.7
	f) Earth transition to pipe	0.5	1.0
ii)	*Closed transition*		
	a) Square or rectangular to round (maximum angle with centerline = 7.5°)	0.1	0.2

The maximum angle between the water surface and the centreline should not exceed 27.5° for inlet transitions and 22.5° for outlet transitions for the best hydraulic conditions, provided velocities are 3 m/s or less. In some structural design, it may prove economical to use 25° to allow the same structure to be used for both inlets and outlets. A 30° angle is often used on inlet transitions with checks, in which case an additional loss is allowed for the checks. The maximum angle is 7.5° for tunnels or if velocities exceed 3 m/s.

NOTE — The broken back refers to the intersection of the vertical and sloping plane surfaces on the sides of the transition.

8.6.4 *Loss of Head Due to Skin Friction in the Barrels and Troughs*

8.6.4.1 The loss of head due to skin friction in the barrels 'h_4' may be computed in accordance with the procedure given in IS 2951 (Part 1).

8.6.4.2 Loss of head due to friction in troughs h_4 should be calculated by the Manning's formula, namely:

$$v = \frac{1}{n} R^{2/3} S^{1/2}$$

where

v = mean velocity, in m/s;
R = hydraulic mean radius, in m;
S = slope; and
n = Manning's constant.

For uniform channel sections covered with sand and gravel, the equivalent Manning's '*n*' may be determined by the Strickler equation as given below:

$$n = \frac{d_{50}^{\ 1/6}}{21.1}$$

where

d_{50} = particle size in which 50 percent of the bed material by weight is finer, in m.

To choose the value of '*n*' *see* IS 2912. Depending upon the smoothness, hardness and rendering of surfaces of the structure (concrete, plaster or masonry, etc), planeness, workmanship and quality control the value of rugosity coefficient *'n'* may be reduced for design purposes from the typical value, so as to achieve reliability in head loss estimates.

Manning's constant depends upon the characteristics of the material and the surface roughness. In absence of actual investigation or established norms being available, value of Manning's constant as given in Table 5 may be assumed.

8.6.5 *Effect of Channel Sinuosity*

Flow resistance in concrete lined canals generally increases with channel sinuosity. Canal structure piers located in the flow prism causes significant increase in water depth, especially in canals having very flat slope. To minimize losses, curves should have a minimum radius of 3 times the top width of canal at TBL, when the velocity is less than 1 m/s. For velocities greater than 1 m/s, consideration should be given to longer radius.

8.6.6 *Checks*

A loss equal to 0.5 times the difference in velocity head between the check opening and the upstream canal section is usually adequate, but more accurate computations may be required where head is critical. Use 0.03 m minimum loss for isolated checks in small canals and 0.015 m in large canals where check structures are provided with streamline transitions. About 1.1 m/s is the maximum velocity through check structures using stop planks, owing to difficulty in operation. A velocity of 1.5 m/s is not objectionable through most structures using gates.

8.6.7 *Flumes (Measuring Structures)*

To determine the total energy loss over a structure, it is recommended to study separately the parts of the structure, where energy is lost for different reasons. These parts and related losses are:

Table 5 Values of Manning's Coefficient
(*Clause* 8.6.4.2)

Sl No.	Type of Surface Material	Value of *n*	
		Range	Normal Design Value
(1)	(2)	(3)	(4)
i)	*Concrete*:		
	a) Hard, smooth finish, troweled	0.011-0.014	0.013
	b) Float finish	0.013-0.015	0.015
	c) Unfinished (not properly floated)	0.015-0.022	0.020
	d) Neat cement smooth finish	0.01-0.012	0.012
	e) Steel shuttering finish	0.012-0.018	0.016
	f) Wooden planks shuttering finish	0.014-0.020	0.018
	g) Large panel plywood shuttering finish	0.013-0.016	0.016
	h) Large panel smooth form finish, rich concrete (M 30 grade and above) with joints grounded smooth, and all defects rectified	0.011-0.014	0.013
	j) Gunited (rough job)	0.018-0.025	0.022
	k) Gunited (good plane job)	0.016-0.023	0.019
	m) Smooth concrete surface with epoxy or polymer treatment	0.009-0.012	0.011
ii)	*Masonry*:		
	a) Glazed brick, good workmanship very fine joints	0.011-0.015	0.013
	b) Good quality brick masonry in cement mortar	0.012-0.018	0.018
	c) Rubble masonry in cement mortar	0.017-0.030	0.025
	d) Dry rubble	0.023-0.035	0.032
	e) Dressed ashlar masonry	0.013-0.018	0.016

a) The losses between the upstream head measurement section (gauging station) and the control section in the throat. Here energy losses occur mainly due to friction and acceleration of flow.
b) The losses due to friction between the control section and the section where the flow head at downstream referenced to weir sill could be measured.
c) The losses due to incomplete conversion of kinetic into potential energy over the downstream transition.

The head loss through flume can be calculated using the following formula:

$$\Delta H = \frac{Lv^2}{R}\frac{n^2}{{C_u}^2 R^{1/3}}$$

where

ΔH = head loss, in m;

L = length in direction of flow, in m;

R = hydraulic radius, in m;

N = Manning's roughness coefficient;

C_u = units coefficient for the Manning 'n'; and

v = average flow velocity, in m/s.

8.6.8 *Hydraulic Jump*

The hydraulic jump is an abrupt rise in water surface that occurs in open channel when water flowing at supercritical flow is retarded. It occurs frequently in a canal down stream from a regulating gate, or at the place where a steep channel slope suddenly flattens, such as at the bottom of a chute. If the jump is small, the change in depth is small, the water will not rise obviously and abruptly but will pass from the low to the high stage through a series of undulations gradually decreasing in size. When the jump is large, it is very obvious and abrupt and it involves a relatively large amount of energy loss through dissipation in the turbulence in the jump.

The depth of flow before the jump is called the initial depth d_1, and that after the jump is called sequent depth d_2.

The specific energy of the flow entering the jump is:

$$E_1 = d_1 + V_1^2/ 2g$$

The specific energy of the flow leaving the jump is:

$$E_2 = d_2 + V_2^2/ 2g$$

From these equations, the energy loss or head loss in hydraulic jump can be expressed by

$$\Delta E = E_1 - E_2 = \frac{\left(d_2 - d_1\right)^3}{4\, d_1 d_2}$$

8.7 Transition Walls

Transition walls as seen in plan, should at their ends, turn nearly at right angles to the flow in the channel and should extend for a minimum length of 0.6 m into the earth bank. Suitable pitching may be provided to the slopes, beyond the transition end.

8.8 Fluming Ratio

Except when dictated by conditions particular to a specific structure, a fluming ratio less than 70 percent may not be adopted. For the purpose of computing the fluming ratio of canal, the width at mid depth may be taken as one hundred percent. In drainage channel when the course is undefined, a fluming ratio from 70 to 90 percent of the Lacey's waterway may be adopted.

8.9 Structure and Earth Work Connection

The earth mass in vicinity of the rigid structures is the connection between rigid structure and flexible earthwork. The rigid structure is non settling, relative to the earthwork. The deflections, settlements and other movements in the rigid structure are comparatively very small. The rigid structure may consist of masonry, PCC, RCC, etc. The connection between rigid structures and earthwork is to be designed so as to reduce the differential settlement, and to avoid the possibility of formation of a separation (cleavage) between the two. The condition of connection between the rigid structure and the earthwork affects the seepage, creep coefficient and piping and thus affects the stability of the earthwork.

For the connection, soil of proper qualities should be chosen. The method and the amount of compaction should be as required.

8.9.1 The canal embankment adjoining the cross drainage structure should have adequate provisions to avoid possibility of any breach and to minimize seepage. The outer slope of the embankment should have a clear cover of 600 mm over the designed phreatic line (*see* IS 7894) for the worst combination of design flood in natural drainage channel and annual low water level.

High earth banks (say over 5 m above ground) should be checked for stability of slopes and provision of rock-toe with filter should be made. Rip-rap or pitching should be done up to a level 0.5 m above HFL plus afflux as applicable (*see* IS 8237, IS 10751, IS 11532 and IS 12094). For large drainage channel properly designed guide banks may be required.

8.9.2 The water flow through various soil strata should be engineered. Flow net through earth work and foundation strata is to be estimated. Exit gradient of seepage water should be limited within the permissible limit.

Adequate foundation depth or cut-off or curtain walls may be provided of suitable depth so as to get safe exit gradient, which may be worked out in accordance with Khosla theory for two dimensional flow. In large structures three dimensional seepage flow may be considered for estimating exit gradient.

8.9.2.1 The permissible creep coefficient (head loss per unit length) through soil may depend upon degree of compaction, whether it is refilled, actions at soil structure connection or interface, relative movements between soil and rigid structure, probable settlements, etc.

Generally the permissible value of exit gradient for flow through different types of soils can be adopted as below:

a) Clay : 1 in 4
b) Shingle : 1 in 4 to 5
c) Coarse sand : 1 in 5 to 6
d) Fine sand : 1 in 6 to 7

8.9.2.2 At entry end of seepage path, an allowance, as discount, should be assumed for settlement and inefficient maintenance, separation, cracking, etc. At exit end a discount or allowance should be assumed against erosion, scour or settlement. In absence of an estimate, the allowance may be 0.6 m high at entry point and 1 m at exit point. This allowance in height is the length over which creep coefficient is to be assumed as zero.

8.9.2.3 From the plot of flow net, the differential design pressure (soil and hydraulic) on the rigid structure can be estimated.

8.9.3 The bottom slabs of box (or barrel) or culverts should be checked for safety against the uplift force in a severe combination of forces chosen. Checks should consist of,

a) safety against movement or flotation during construction and in service; and further
b) check for design stresses with severe combination of uplift.

8.9.3.1 For safety against flotation or movement, if required, the gravity loads can be increased by increasing thickness of members, providing additional concrete or masonry for weight, or anchoring the members into the foundation strata or deeper. The sum of downward equilibrating forces should be at least 1.2 times the upward buoyancy force. For checking the stress condition in the members, the dead loads or the downward equilibrating force (such as anchorage) should be reduced by dividing these by 1.2. Anchorages and parts of anchorage system should be checked for stresses under full load required for safety against floating.

8.9.4 When flow is through multi-barrel, at the upstream and downstream ends, stop-log grooves may be provided by extending the partition wall, so as to facilitate isolating one or more barrels for maintenance or repair. Ends of the partitions of multi-barrels should be provided with cut and ease water shapes so as to minimize the energy loss.

8.10 Abrasion Resistance

Structural members in the bed of the flow should be safe against abrasion loss. Depending on the velocity of flow and the abrasion causing debris (stone gravel, sand silt, etc) it may hold, the structural member in the bed should be hard enough and of sufficiently high strength or it should be given a treatment or wearing coat to enhance the abrasion resistance. Fitching of heavy weight stones, stone masonry or high grade concrete overlays may be provided.

Apart from the bed of flow, the members on the sides may also be subjected to abrasion, though the material causing abrasion may be less effective on the sides than at the bottom. Hence due consideration for durability of sides, along with the flow should be given.

9 FOUNDATION

9.1 Foundations of a cross drainage work should be designed to satisfy the requirements of allowable bearing capacity of the foundation strata under critical loads including positive pressure conditions (that is no uplift or tension), seismic effects, anticipated scour and settlement.

9.1.1 The sub-grade at the foundation level for known shape and size of foundations, depth below ground level, expected sub-soil water level and engineering properties should be safe in bearing capacity. Comparatively lower pressures are allowed on foundations on sub-grades prone to appreciable settlement.

9.1.2 As far as possible, the foundation should bear on homogeneous, undisturbed and uniform sub-grade of fairly dense type. Where foundations have to be provided on sub-grade of different types suitable joints should be provided to avoid cracks due to differential settlement, the tolerance limit has to be evaluated for each case.

9.1.3 The permissible maximum differential settlement of the foundation strata estimated should not exceed 1 in 400. In case of structures sensitive to differential settlement, the tolerance limit has to be evaluated for each case.

9.1.4 When the seismic effect is considered, higher bearing capacity may be adopted in accordance with IS 1893.

9.1.5 The foundations should be taken sufficiently deep to secure firm strata from considerations of settlement, overall stability and avoidance of undermining due to erosion. The depth of foundation of various members should be such that these are safe against scour or are protected against it. If sound rock is met with, at the higher levels than the anticipated scour, scour criteria shall not be applicable. Tension (that is negative contact pressure) can be allowed only, if foundation strata consists of hard rock, however, such negative contact pressure should be neglected in the design calculations.

9.1.6 Maximum depth of scour should be computed for stream as in **8.5** from check high flood level (CHFL). Either the foundation or cut-off wall if provided, under the foundation should be taken to a depth 1.333 times the scour depth estimated below CHFL or 1.05 times the scour depth estimated for probable maximum flood (PMF). Either depth of foundation, or depth of cut-off if provided, should also be governed by permissible gradient of seepage water as in **8.9.2**. In case of canal having erodable bottom (that is unlined) similar checks should be done for full supply discharge.

9.1.7 Where concrete or masonry floor is provided under the works, scour condition is not applicable and the foundations are usually taken to about 1.5 m below the floor levels with suitable cut-off for the concrete or masonry floors. However where such a floor is not provided, foundations are taken to provide a margin below the anticipated scour level (usually called grip length) of about 0.33 times the maximum depth of scour.

10 MODEL STUDIES

Owing to a number of complex factors in the design of hydraulic structures and specially when the designs are based on empirical formulae, adequate answers cannot be obtained through analytical methods only. Therefore, it would be in the best interest, if the designs for major cross drainage works are first tested on models. These studies should *inter alia* taken into account the impact of ancillary components of the structure, namely, the approaches, end connections, afflux bunds, floors, protection arrangements and any upstream or downstream structures on either the canal or the drainage channel.

11 MISCELLANEOUS DETAILS

11.1 Waterstops

Waterstops, also referred to as water seals, are generally of three types, namely: (a) Rubber water seals, (b) Metal water seals, and (c) Synthetic material seals. The waterstops are used in and across all joints where leakages are detrimental to structural safety or the water needs are to be conserved. The locations where waterstops are provided in various types of cross drainage works are described below:

a) *Aqueduct* — In R.C.C. through side walls and bottom slab over each pier in a continuous length and at the junction of transition and R.C.C. trough, both in the floor and wing walls.
b) *Syphon* — At expansion joints and at the junction of each of the sloping limbs in a continuous form and at the junction of the transition walls and floors with the barrel, both at the entry and exit in a continuous form.
c) *Super passage* — At the junction between the drainage trough wing walls, namely, trough wall of R.C.C. and wing wall of masonry and all the expansion joints in a continuous length.

11.2 Weep Holes

Weep holes are small openings in the retaining walls, like wings (that is transitions of natural stream). These are to facilitate the drainage of backfills and avoid build up of pressure. Weep holes may be provided above the flow net line of zero water pressure, under the condition of canal flowing full and natural stream with lowest annual flow.

Weep holes if provided, should have filters with graded material suitably provided to avoid piping of earth fill behind the wall and also to avoid choking of the holes.

The provision of weep holes should be so, as to not render the creep coefficient of seepage unsafe, and should also not contribute to enhanced loss of canal water.

11.3 Bearings

For safe transfer of load from superstructure to sub-structure suitable bearings should be provided between the trough bottom and pier/abutment to cater for the various movements occurring in the superstructure under different combinations of load.

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This Indian Standard has been developed from Doc No.: WRD 13 (0447).

Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

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Published by BIS, New Delhi